PRELIMINARY GEOTECHNICAL INVESTIGATION
NEWLAND SIERRA
SAN DIEGO COUNTY, CALIFORNIA

Prepared For:

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Project No. 10618.002

June 29, 2015
(Revised November 6, 2015)

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

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To: Newland Sierra, LLC
9820 Town Centre Drive, Suite 100
San Diego, CA 92121

Attention: Ms. Rita Brandin

Subject: Preliminary Geotechnical Investigation, Newland Sierra, San Diego County, California

In accordance with your request, we have performed a preliminary geotechnical investigation for the Newland Sierra project located in northern San Diego County, California. This report presents results of our preliminary geotechnical investigation and provides a summary of our conclusions and recommendations relative to the proposed development within the roughly 2,000-acre site and utilizes current conceptual grading plans as a basis for presenting our conclusions and recommendations. It should be noted that geologic review, geotechnical observations, and laboratory test results completed as part of the previously proposed Merriam Mountains Property project were incorporated into various sections of this report and are considered pertinent and applicable to the current proposed project. As plans become finalized, some additional recommendations may become warranted. The site is to be developed with a number of localized development areas connected by roadways while preserving large areas of open space, steep slopes, and wildlife corridors. A final grading plan review should be performed prior to the start of construction.

Based on the results of our geotechnical investigation, the proposed development of the site is considered feasible from a geotechnical standpoint provided the recommendations summarized in this report are implemented during the final design, grading, and construction phases of the development. This report includes the current grading plans with our geotechnical map.

If you have any questions regarding our report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

In accordance with your request and authorization, this report presents the results of our preliminary geotechnical investigation for the roughly 2,000-acre site in the northern area of San Diego County, California (Figure 1). The purpose of our preliminary investigation was to evaluate the pertinent geotechnical conditions at the site and to provide preliminary design criteria for the proposed development. This report is based on the current grading plan (Fuscoe, 2015) which is utilized as a base map for the Geotechnical Map (Plates 1 and 2).

This report incorporates and expands on previous preliminary studies for the Merriam Mountains Property (Leighton 2000, 2002, 2004, 2005a, 2006 and 2007). The investigation presented herein utilizes field investigations and laboratory testing to provide conclusions and recommendations relative to the geotechnical conditions at the site based on current grading plans.

The scope of services for our preliminary geotechnical investigation included:

- Review of pertinent available geotechnical literature (including previous geotechnical reports), geologic maps, and aerial photographs (Appendix A).

- Geologic mapping of the site. This field mapping included identification and mapping of bedrock units and extent of surficial soil deposits and also areas of perched granite boulder outcrops. The site geologic conditions are shown on the Geotechnical Map (Plate 1).

- A subsurface exploration program consisting of the excavation, sampling and logging of 42 shallow exploratory trenches excavated in 2002 and 6 additional trenches (trenches T-43 to T-48) excavated in 2004. In addition, 52 trenches (track excavator), 3 hollow-stem auger borings, and 44 rock drilled (air-track) borings were excavated, sampled, and logged in 2007 as part of a rippibility study (Leighton, 2007). The trenches were excavated to evaluate the characteristics of surficial soils and bedrock materials. Logs of the trenches and borings are presented in Appendix B. The approximate locations of the trenches and borings are presented on Plates 1 and 2.

- Laboratory testing of representative soil samples. Laboratory test results are included in Appendix C.

- Percolation testing at two locations.

- Seismic refraction studies were performed to evaluate the approximate seismic velocities of the rock material within anticipated excavation areas in order to provide a rough estimate of the rippability characteristics of the materials. The approximate locations of the seismic traverses are presented on the Geotechnical Map (Plates 1 and 2).

- Evaluation of site seismicity.

- Review previous slope stability analysis of proposed cut and fill slopes.

- Preparation of this preliminary geotechnical report presenting the findings of our study and providing conclusions and recommendations relative to the currently proposed development.

1.2 Site Description

The property is generally located west of Interstate 15, east of North Twin Oaks Valley Road, Deer Springs Road to the south and Lawrence Welk Lane to the north, in an area of San Diego County named the Merriam Mountains (Figure 1). Topographically, the site generally consists of moderate to steeply sloping hillside terrain with localized valleys and gently sloping terraces. Elevations range from a high of approximately 1,765 ± feet mean sea level (msl) in the west-central portion of the site to a low of 800 ± feet (msl) along the southerly site boundary.

Generally, natural drainage is presently accomplished through a network of narrow steep-sided canyons in all directions away from the approximately central, northwesterly trending ridgeline. The largest canyon on the site is located along the southerly site boundary and drains in a southward direction. Vegetation on the site ranges from native grasses and weeds in the relatively flat areas canyon bottoms to moderate to thick chaparral on the upper elevations.

Man-made features on the site include: 1) two above-ground water storage tanks and associated buried water lines; 2) short sections of deteriorating asphalt pavement access roads adjacent to the water tanks: 3) numerous dirt roads which cross the property; 4) minor amounts of undocumented fills associated with the dirt roads and water tank pads; 5) an abandoned crude runway within an elevated valley in the west portion of the site; 6) fences around most of the site perimeter and scattered fences within; and 7) an abandoned rock quarry at the west margin of the property and outside of the areas proposed for development. Existing cut slopes within this abandoned quarry are as high as 225 feet with gradients steeper than ½:1 (horizontal to vertical). It is also possible that several water wells are present on site.

Site Latitude and Longitude
33.2156º N
117.1489º W

1.3 Proposed Development

The current plan proposes to develop a master-planned community integrating residential, commercial, recreational and open space land uses. The proposed project will consist of 2,135 single-family, multi-family, and variable residential dwelling units with an overall density of 1.08 dwelling units per acre within the 1,985 acre area. Proposed residential products range from attached units to 7,000 square foot single family lots. In addition, the project would also include neighborhood commercial uses, parks, trails, open space areas and associated community facilities and infrastructure.

It is anticipated the localized development areas will be primarily located in the flatter site areas with cuts in the higher elevations and fill areas anticipated in lower site areas. Cut and fill slopes were originally planned at 2:1 (horizontal to vertical) gradients. In order to minimize intrusion into open space areas, reduced slope heights and overall grading, both cut and fill slopes are planned to be at inclinations of 1-1/2:1 (horizontal to vertical) and with 1:1 cut slopes along proposed roadways. The maximum height of proposed fill slopes is on the order of 186 feet while the maximum height of proposed cut slopes is on the order of 85 feet. Although not recommended at this time, steeper slopes may be locally feasible in rock slopes as demonstrated by the existing slopes in the area of the abandoned quarry. However, a site specific evaluation of the area and proposed slope will be required prior to design.

Foundation recommendations are not included as part of this report, but will be made available once final development plans, building types, and locations are finalized.

Deer Springs Road is anticipated to be widened south of the project area. Proposed grading and improvement plans for this widening are not yet finalized. Additional geotechnical recommendations can be provided upon review of proposed roadway plans.

1.4 Surface Investigation and Laboratory Testing

Our initial subsurface investigation in 2002 consisted of the excavation, logging and sampling of 42 exploratory trenches to a maximum depth of 17 feet. Subsequently, in 2004 six additional trenches, T-43 through T-48, were excavated after preliminary site development plans became available and grading of the southeast portion of the site was proposed. In 2007, the excavation of three small diameter borings (BH-1 through BH-3) were performed to depths of 20 feet below the existing ground surface to evaluate the physical characteristics and engineering properties of the alluvial soils within the main canyon along the southwestern entry to the site. Note that in this canyon, accessed from Sarver lane to the south, we have mapped the most significant deposits of alluvium, in an area primarily designed to receive fill based on the proposed fill placement and higher proposed grades. The boring excavations allowed evaluation of the soils within the broad canyon, which were observed to be loose and porous in the upper portion, based on our trench excavations. Representative bulk and relatively undisturbed (drive) samples were obtained at frequent intervals for laboratory testing.

Recently, four shallow borings were advanced to depths ranging from 6 to 7 feet below the existing ground surface in the vicinity of the two proposed storm water detention basins to perform field percolation testing. The approximate locations of the borings with the field percolation tests (FP-1 through FP-4) are presented on Plate 1 and field percolation test data is presented in Appendix B.

Note that all of the trenches and borings were logged by our geologist and logs are presented in Appendix B. Approximate locations of the trenches and borings are shown on the Geotechnical Map (Plates 1 and 2). Subsequent to the subsurface investigation and logging by our geologist, the trenches and borings were backfilled.

For the 2007 rippability study, we supplemented our initial 2002 subsurface exploration with 52 “Excavator Trenches” (T-49 through T-100), 44 Rock Drill or Air-Track borings (AT-1 through AT-44) and 6 additional seismic traverses (S-13 through S-18, performed in the southern and eastern portions of the site). The approximate locations of the explorations are shown on Geotechnical Map (Plates 1 and 2). Note that for comparison purposes, an excavator trench, seismic line, and air-track boring were advanced side by side in several locations. Details of the rippability study exploration are discussed below.

Excavator Trenches

The trenches (T-49 through T-100) were excavated utilizing a Hitachi EX450 track-mounted excavator using a 30-inch bucket equipped with rock teeth. The track-mounted excavator was utilized to visually observe the extent of rippable granitic bedrock and to correlate data from adjacent hydraulic percussion rock drill holes, as well as seismic refraction lines. The trenches were excavated to practical refusal where the full effort of the excavator could not appreciably advance into the hard rock at the trench bottom. Representative photographs of these excavations are included in Appendix B. The logs exploratory trenches performed with the excavator are included in Appendix B.

Rock Drill Borings

The rippability study also utilized a hydraulic percussion-type rock drilling rig to aid in the evaluation of the rippability characteristics of the granitic bedrock. The hydraulic percussion rock drill rig uses drill advancement rates to determine relative densities of the subsurface materials. The borings were advanced to a maximum depth of 57 feet utilizing an Ingersol Rand ECM-370, track-mounted, hydraulic powered drill with a 4-inch diameter carbide bit obtained from Tom C. Dyke Drilling and Blasting. The advancement rates provide an approximation of the depth to the rippable, marginally rippable, and non-rippable rock.

The drill down pressure and rotation speed was maintained consistently during the drilling operations. The borings were commonly advanced until several feet of non-rippable granitic bedrock was encountered. The borings were advanced through the design elevations (i.e. finish grade elevations of the proposed lots) where feasible. Generally, drill rates slowed considerably before the design elevations could be achieved, indicating most of the deeper cut areas contain significant thicknesses of non-rippable rock. The logs from the hydraulic-percussion borings are provided in Appendix B.

Seismic Refraction Survey

To supplement our original geophysical study (S-1 through S-12), six additional seismic refraction lines performed in the south and east portions of the site (S-13 through S-18). The purpose of these surveys performed by Subsurface Surveys, Inc., was to evaluate the approximate seismic velocities of the native materials in order to provide further estimates of the rippability characteristics of the site. The results are presented in Appendix D and discussed in Section 4.2. The approximate locations of the traverses (S-1 through S-18) are presented on the Geotechnical Map (Plates 1 and 2).

Field Percolation Testing

Four field percolation test borings were performed to evaluate the existing on site soils for potential infiltration of storm water (i.e., two at the proposed basin north of Gist Road, and two at the proposed basin north of the Deer Springs and Mesa Rock intersection). Depths of the hand augured borings for the tests were approximately 6 to 7 feet below the existing ground surface.

Laboratory testing was performed on samples obtained during the excavation of the exploratory trenches and small diameter borings. Testing has included in situ moisture and density, laboratory maximum dry density and optimum moisture content, shear strength and hydro-consolidation. The results of the data are presented in Appendix C.

2.0 GEOLOGY

2.1 Regional Geology

The site is located within the coastal subprovince of the Peninsular Ranges Geomorphic Province, near the western edge of the southern California batholith. The topography at the edge of the batholith changes from the rugged landforms developed on the batholith to the more subdued landforms, which typify the softer sedimentary formations of the coastal plain. Primarily, the site is underlain by the Cretaceous-aged Granite of the southern California batholith with minor amounts of Jurassic-aged metavolcanic rock along the western margin (see Figure 2 – Regional Geology Map). Erosion and regional tectonic uplift created the valleys and ridges of the area.

2.2 Site-Specific Geology

Based on our site visit and review of our referenced geologic maps (Appendix A), the primary bedrock unit onsite is Cretaceous-aged Granite although Jurassic-aged Metavolcanic rock is present along the western margin. These units are in turn overlain by surficial units consisting of colluvium, alluvium, slopewash and minor undocumented fill soils. Surficial soil deposits generally consist of relatively fine-grained material useful during grading of a site where abundant oversize material is expected. A brief description of the geologic units encountered on the site is presented below and their approximate aerial extent is provided on the Geotechnical Map (Plates 1 and 2).

2.2.1 Undocumented Fill Soils (Afu)

Undocumented fill soils were observed in a number of places on the site. As observed, the undocumented fill soils were generally associated with the grading of the onsite dirt roads and water tower pads. In general, these undocumented fill soils were found to be relatively limited in extent and are therefore unmappable at the scale of this investigation. Significant deposits of artificial fill were mapped at the western property margin, associated with the previous quarry operation located there. All existing undocumented fills located on the site are considered potentially compressible and unsuitable in their present state for structural support.

2.2.2 Topsoil/Colluvium (Unmapped)

The topsoil/colluvium observed during our field study mantles the mid- to lower-portions of the hillsides across the majority of the site. The topsoil/colluvium consists of light brown to brown, damp to moist, loose to medium dense, silty to clayey, fine to very coarse sand. These soils are typically massive, porous and contained scattered roots and organics. The potentially compressible topsoil is estimated to be approximately 0 to 2 feet thick. Localized areas of thicker accumulations of topsoil may be encountered. On hillsides of higher elevation, topsoil is minimal, as shown in Figures P1 and P2. Topsoil/colluvium soils on the lower hillsides of the onsite drainages can be expected to be somewhat deeper in extent and locally variable in composition. Topsoil and colluvium were not mapped due to the relatively thin exposures although locally thicker soil/colluvial profiles were encountered in the flatter areas in the southeast portion of the site. The seismic profiles (Appendix C) identify a “topsoil/colluvium” layer averaging 4.5 feet in thickness. We generally interpret this to include a thin weathered profile of granitic rock, and map it as such (Kgr below). This material should be completely removed and recompacted in areas of proposed development.

2.2.3 Alluvium (Map Symbol – Qal)

Quaternary-aged alluvium is present in the bottom of the canyons and drainages on the site. Similar to colluvial deposits, these soils are generally thin (less than 3 feet) and unmapped, except where identified by our subsurface investigation and field mapping. The areas identified as containing alluvial soils have been mapped on the Geotechnical Map (Plates 1 and 2). Significant thicknesses of alluvial deposits have been identified within the following areas: 1) the northwestern canyon near the crude abandoned runway (3 to 10 feet); 2) the northeast-southwest trending canyon within the southwestern portion of the site (6 to 10 feet within Trenches T-30 and T-31), and 3) the main canyon (Figure P3) accessed by Sarver Lane to the south of the site (over 10 feet).

It should be noted that alluvial soils likely underlie all of the onsite canyons, but in the upper elevations of the site, the canyons were considered to be too narrow in lateral extent to be presented on the map at the scale provided. These soils typically consist of brown, damp to wet, loose to medium dense/stiff, silty sands, sandy clays and silty clays. The alluvium is also considered to be moderately porous and usually contains localized zones of moderate to abundant roots and other organic matter. Dry, porous, and compressible alluvium is not suitable for support of additional fill and or structural loads and should be completely removed and recompacted in areas of proposed development.

2.2.4 Older Quaternary Alluvium (Map Symbol - Qalo)

Older alluvium was encountered in the deeper portion of the main canyon in the southwest corner of the site (Borings BH-1 through BH-3). It consisted of red-brown to orange-brown silty sand with gravel older than the overlying Quaternary alluvium (Qal), the older alluvial deposits are generally medium dense to dense and moist. These materials are mapped in the lower portions (deeper than 10 to 15 feet bgs) of the main canyon accessed by Sarver Lane. They are expected to represent the deeper, relatively dense and moist alluvial deposits. Subsequent to the removal of the overlying younger alluvial deposits, the older alluvium should be removed to moist, dense, competent material, as determined by the geotechnical consultant in the field (generally 10-15’ bgs).

2.2.5 Quaternary Slopewash (Map Symbol – Qsw)

Quaternary slopewash includes residual materials shed from slopes and deposited on the lower portions of the slopes and within localized drainages. As encountered, the materials consist of light brown to gray-brown silty sand, dense and generally homogeneous, as shown in Figures P4 and P5. Resistant clasts of relatively unweathered granite are locally suspended within the deposits which we anticipate to be on the order of 5 to 20 feet in thickness. These deposits are generally medium dense to dense, but are still locally porous and potentially compressible. This unit is not suitable for the support of additional fill or structural loads and will require removal and recompaction prior to fill placement.

2.2.6 Possible Quaternary Landslide Deposits (Map Symbol – Qls)

Our review of available geologic literature indicates the presence of possible ancient landslide debris on the eastern edge of the site. This possible large landslide was mapped by others along the central portion of the eastern site property line and beneath Interstate 15 (SDAG, 1988). The landslide has only been mapped based on its surficial expression and has never been confirmed by a subsurface investigation. Based on our understanding of the proposed project, the mapped landslide does not appear to be in the vicinity of any of the areas of proposed development. If the site development plans are changed to include development of these areas of the site, or if landslide materials are encountered in subsequent subsurface investigations of the site or during site grading, remedial measures may be necessary.

2.2.7 Cretaceous Granitic Rock (Map Symbol – Kgr)

Granitic rock outcrops were observed across the vast majority of the site and granitic rock underlies the site at depth where not exposed at the surface. The material generally consists of medium to coarse-grained quartz-rich granite rock. Large granitic boulders characterize the outcrops in the upper regions of the site, while in the mid- to lower-regions of the site; weathered granitic material was observed in road cuts below the topsoil/colluvium. The weathered granitics generally consists of light gray to light red brown, damp, dense, fine to coarse sand with localized residual boulders throughout. As observed in our trenches and existing road cuts, the depth of highly weathered rock varied from inches to less than 1 to 7 feet below the ground surface, or bedrock contact where these materials are buried. The seismic traverses performed at the site and observation of existing cut slopes also substantiates the very dense nature of the onsite bedrock. Due to the dense nature of the rock, heavy ripping and/or blasting is highly likely in cut areas deeper than 5 to 10 feet. For further discussion of rippability, see the results of our Rock Drilling and Seismic Refraction Study in Section 4.2.

Localized areas containing abundant granitic boulders located on oversteepened, elevated promontories, have been identified across the site and in particular in the steep areas in northern portions of the site. Perched boulder outcrops, (as shown in Figures P6 and P7) if located directly above proposed development areas, may require mitigation measures as discussed in Section 6.7.2.

2.2.8 Jurassic-Cretaceous Metavolcanic Rock (KJm)

Metavolcanic rocks are mapped within a narrow band along the western margin of the subject site, including the quarry area. This relatively variable unit consists of schist, quartzite, argillite, gneiss, and meta-basalt (Tan and Kennedy, 1996). These metamorphosed rocks represent the older rocks, intruded and altered by the young Cretaceous-aged Granitics (Kgr), and tend to be more basic, and therefore less resistant to weathering. These rocks generally form more subdued erodible topography than the adjacent Cretaceous granitic rocks mapped to the east and west. Based on field observations; the upper portions of this material is expected to be relatively rippable in the near surface but will likely become difficult to excavate at depth or where previously mined. No subsurface exploration was conducted within this unit as part of this investigation. Additional investigation of this unit may be warranted if major cuts are proposed in this area.

2.3 Geologic Structure

Based on our subsurface investigation and site reconnaissance/geologic mapping, the materials on site are generally massive with no distinctive structure. Some regional foliation and/or fracturing were observed during our aerial photographic analysis. Subsequent field mapping has identified jointing in localized areas within the weathered granitic rock at the surface (Figure P8). Fracture fabric trends, as well as joint orientation attitudes, are shown on the Geotechnical Map (Plates 1 and 2). In general no adverse structural conditions were noted during this study.

2.4 Surface and Groundwater

No surface or shallow groundwater conditions were encountered during our field investigations of the site. However, surface flow is anticipated in the onsite drainages after periods of heavy rainfall. In addition an area of possible seepage has been reported in the northern section of the site near the abandoned dirt airfield. Our trenches in this area did not encounter any zones of seepage; however, it may be present in wetter years.

Near surface groundwater seepage should be anticipated at the topsoil/bedrock contact after periods of heavy rainfall. It is anticipated that ground water levels will fluctuate during periods of high precipitation and or irrigation and may become perched on the underlying bedrock or concentrated in fractures within the bedrock. Localized seeps may occur after period of heavy rainfall or irrigation in cut areas along fractures and/or joint systems. Recommendations for mitigation of groundwater concerns include a series of canyon subdrains as well as possible toe of slope drains, stability fills, and/or underpad drains. Treatment of possible seepage within building pads or cut-slope areas can be provided on an individual basis after evaluation by the geotechnical consultant during grading operations.

2.5 Mineral Resources

The site is located in an area where mineral resources are known to exist and an abandoned quarry is present in the northwestern portion of the site. The approximate location of this abandoned quarry can be seen on Plate 2, Geotechnical Map. We note that the abandoned quarry is outside of the area of proposed development.

Review of published maps of the Geology and Mineral Resources of San Diego County, California (Weber, 1958) indicates the mineral resources consist predominately of rock that is quarried for sand, gravel and aggregate purposes. These types of mineral resources are typical for much of northern San Diego County.

2.6 Infiltration

In summary, we performed four field percolation tests (two at the proposed basin north of Gist Road, and two at the proposed basin north of the Deer Springs and Mesa Rock intersection) to evaluate the existing on site soils for potential infiltration of storm water. Locations of the tests are presented on Plate 1 of the Geotechnical Map. Results of the field percolation tests indicate that the site soils have a percolation rate ranging from approximately 4.6 to 14.7 minutes per inch (mpi). Note that the designer of the onsite storm water infiltration system should consider the results of field testing, which may vary by 10 percent, and use engineering judgment in developing an appropriate system. In addition, based on our experience, we anticipate that once the site has been graded for the proposed development the underlying compacted fill consisting of a mixture of soils will have permeable and impermeable layers can transmit and perched ground water in unpredictable ways. Therefore, Low Impact Development (LID) measures may impact down gradient improvements and the use of some LID measures may not be appropriate for this project. All Infiltration and Bioretention Stormwater Systems design should be reviewed by geotechnical consultant.

3.0 FAULTING AND SEISMICITY

3.1 Faulting

Our discussion of faults on the site is prefaced with a discussion of California legislation and state policies concerning the classification and land-use criteria associated with faults. By definition of the California Mining and Geology Board, an active fault is a fault which has had surface displacement within Holocene time (about the last 11,000 years). The State Geologist has defined a potentially active fault as any fault considered to have been active during Quaternary time (last 1,600,000 years). This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Earthquake Faulting Zones Act of 1972 and as most recently revised in 1997 (Hart, 1997). The intent of this act is to assure that unwise urban development does not occur across the traces of active faults. Based on our review, the site is not located within an "Earthquake Fault Zone" as identified in this document.

Our review of available geologic literature indicated that there are no known active, potentially active, or inactive faults that transect the subject site (Appendix A). The nearest known active regional fault is the Elsinore-Julian Fault. The closest projected trace for this fault zone is located approximately 12 miles east of the site. The maximum peak horizontal ground acceleration as a result of the maximum credible earthquake is expected to be produced by the Elsinore-Julian Fault located 13 miles east of the site.

3.2 Seismicity

The site is considered to lie within a seismically active region, as is all of Southern California. As previously mentioned above, the Elsinore-Julian Fault zone located east of the site is considered the 'active' fault having the most significant effect at the site from a design standpoint. The Seismic Parameters for Active Faults in the vicinity of the site are presented below.

Seismic Parameters for Active Faults (CGS, 2003)			
Potential Causative Fault	Distance from Fault to Site (Miles)	Slip Rate (mm/yr)	Maximum Magnitude Event (M_w)
Elsinore-Julian	13	5.0	7.1
Elsinore-Temecula	13	5.0	6.8
Rose Canyon	15	1.5	7.2
Newport-Inglewood	16	1.5	7.1
Coronado Banks	31	3.0	7.6

3.3 Seismic Hazards

Severe ground shaking is most likely to occur during an earthquake on one of the regional active faults in Southern California. The effect of seismic shaking may be mitigated by adhering to the California Building Code or state-of-the-art seismic design parameters of the Structural Engineers Association of California.

3.3.1 Shallow Ground Rupture

As previously discussed, no active faults are mapped transecting or projecting toward the site. Therefore, surface rupture hazard due to faulting is considered very low.

3.3.2 Mapped Fault Zones

The site is not located within a State mapped Earthquake Fault Zone (EFZ). As previously discussed, the subject site is not underlain by known active or potentially active faults.

3.3.3 Site Class

Utilizing 2013 California Building Code (CBC) procedures, we have characterized the site soil profile to be Site Class C for the cut areas, and Site Class D for fill areas based on the results of our subsurface investigation.

3.3.4 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Tables 1A and 1B are the spectral acceleration parameters for the project determined in accordance with the 2013 CBC (CBSC, 2013) and the USGS Worldwide Seismic Design Values tool (Version 3.1.0).

Table 1A (Cut Areas)

2013 CBC Mapped Spectral Acceleration Parameters

Site Class	C		
Site Coefficients	F_a	=	1.000
	F_v	=	1.373
Mapped MCE Spectral Accelerations	S_S	=	1.098g
	S_1	=	0.427g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.098g
	S_{M1}	=	0.587g
Design Spectral Accelerations	S_{DS}	=	0.732g
	S_{D1}	=	0.391g

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Table 1B (Fill Areas) 2013 CBC Mapped Spectral Acceleration Parameters			
Site Class	D		
Site Coefficients	F_a	=	1.061
	F_v	=	1.573
Mapped MCE Spectral Accelerations	S_S	=	1.098g
	S_1	=	0.427g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.165g
	S_{M1}	=	0.672g
Design Spectral Accelerations	S_{DS}	=	0.776g
	S_{D1}	=	0.448g

Utilizing ASCE Standard 7-10, in accordance with Section 11.8.3, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.414g for the site. For a Site Class C (Cut Areas), the F_{PGA} is 1.000 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.414g for the site. For a Site Class D (Fill Areas), the F_{PGA} is 1.086 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.449g for the site.

3.4 Secondary Seismic Hazards

Secondary effects that can be associated with severe ground shaking following a relatively large earthquake which include shallow ground rupture, soil liquefaction and dynamic settlement, seiches and tsunamis and seismically induced rockfall. These secondary effects of seismic shaking are discussed in the following sections.

3.4.1 Shallow Ground Rupture

Breaking of the ground because of faulting is not likely to occur on site due to the absence of known faults on the site. Cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

3.4.2 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Both research and historical data indicate that loose, saturated, granular soils are susceptible to liquefaction and dynamic settlement. Liquefaction is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to act as a viscous liquid. This effect may be manifested by excessive settlements and sand boils at the ground surface.

The rock materials underlying the site are not considered liquefiable due to their dense physical characteristics and unsaturated condition. Saturated alluvial soils at the site may have a potential for liquefaction. Our trenches excavated in alluvial areas did not reveal significant thicknesses of saturated alluvium.

3.4.3 Seiches and Tsunamis

Due to the relatively high site elevation, inundation due to a seiche or tsunami is not considered a hazard at the site.

3.4.4 Rockfalls

In general, the site development has been planned to avoid development below natural slope areas of steeply sloping areas with granitic boulder outcrops, where there is a potential for isolated rockfalls to occur. Oversteepened areas of granitic boulders with potential for mobilization have been identified, as shown on Geotechnical Map (Plates 1 and 2), and depicted in Figures P6 and P7. Additional mapping and evaluation of these areas should be performed as plans become finalized and during site grading. Mitigation of rockfill areas could include grading of these areas to remove dangerous boulders, providing support for boulders to remove the hazard, or construction of a deflection/catchment berm or fence.

4.0 GEOTECHNICAL CONSIDERATIONS

Based on the results of our geotechnical investigation of the site and our professional experience on near-by sites with similar soils, the engineering characteristics of the on-site soils are discussed below.

4.1 Expansion Potential

The majority of the onsite soils are expected to have a very low to low expansion potential. Topsoil and alluvium/colluvium soils that occur in minor quantities (in portions of the site) may possess moderate to high expansion potential. Geotechnical observation and/or laboratory testing during grading are recommended to identify areas of highly expansive soils and determine the actual expansion potential of finish grade soils.

4.2 Excavation Characteristics

As part of our preliminary investigation and assessment of the excavation characteristics for the site, seismic refraction field studies were conducted by Subsurface Surveys in 2002 and 2007, which included a total of 18 seismic survey lines. In addition, 44 rock drilled (air-track) borings were excavated in 2007 as part of a rippibility study (Leighton, 2007). In summary, the site is underlain by weathered and unweathered Cretaceous Granitic Rock, capped by variable but generally limited thicknesses of topsoil, undocumented artificial fill, and alluvium. The results of the seismic refraction studies and the rock drilled (air-track) borings are discussed below.

4.2.1 Rock Drill Assessment

In order to group the materials to be excavated in terms of difficulty of excavation, Leighton has adopted a three-fold scheme for the rock drill data. The classification is based on our experience with similar rocks in the San Diego County area and assumes the utilization of a single shank D9R Caterpillar tractor with rock teeth or equivalent equipment and excavation production rates normally acceptable to a contractor. The rippability of the on-site rocks is classified as indicated in Table 2.

Table 2 Rock Drill Rippability	
Drilling Rate	Rippability Classification
Up to 17-seconds per foot of advancement:	Generally rippable
17- to 25-seconds per foot of advancement:	Marginally rippable
Greater than 25-seconds per foot of advancement	Non-rippable

The terms rippable, marginally rippable, and non-rippable refer to general production capabilities of a D9R Caterpillar tractor equipped with a single-shank ripper and rock teeth. These limits are approximate and are subject to fluctuations due to localized variations in rock conditions such as the degree of weathering and decomposition, rock fracture patterns and spacing.

Ripping of rock with higher drilling rates may become totally dependent on time available and the economics of the project. Ripping production will also depend on the skill and experience of the equipment operator. In addition, ripping for scraper loading may call for different techniques other than those used if the same material is to be dozed away or top-loaded. Furthermore, ripping success may depend on the combination of the number of shanks used, length, and depth of shank, tooth angle, direction and speed, etc. We further recommend that the grading contractor perform an independent study to confirm the rippability characteristics for developing production rates based on the type of equipment utilized for the project.

4.2.2 Seismic Refraction Survey Assessment

The purpose of these surveys was to evaluate the approximate seismic velocities of the granite (as close as feasible to proposed cut areas) in order to provide a rough estimate of the rippability characteristics of the materials. Each survey line was approximately 250 feet in length and provided depths of investigation of approximately 75 feet below the existing ground surface. The locations of the seismic survey lines are presented on the Geotechnical Map (Plates 1 and 2).

As described above, a number of air-track borings and excavator trenches were performed across the site, in order to further evaluate the rippability and assess the characteristic of the surficial (rippable) materials in the proposed cut and fill areas. It should be noted that the measured seismic velocities presented represent average velocities of the subsurface materials, and significant local variations, localized hard or cemented zones, concretions, or other anomalies may be present. The air track borings as well as excavator trenches serve to further characterize nature of the granitic rock.

In order to categorize the subsurface materials in the proposed cut areas in terms of excavation characteristics, the following classifications based on the seismic survey results are utilized. This five-fold classification scheme is based on our experience with similar rocks in the San Diego County area and assumes the use of a single shank D9R Dozer (or equivalent). The rippability characteristics of the site materials are classified as follows:

Table 3

Rippability Based on Seismic Refraction Velocities

Calculated Seismic Velocity	General Excavation Characteristic
Up to 2000 feet per second	Easy ripping
2000 to 4000 feet per second	Moderately difficult ripping
4000 to 5500 feet per second	Difficult ripping, possible localized blasting
5500 to 7000 feet per second	Very difficult ripping, possible local to general blasting
Greater than 7000 feet per second	Blasting required

“Difficult ripping” refers to rocks, in which it becomes difficult to achieve tooth penetration, sharply reducing ripping production. Local blasting may be necessary in order to maintain a desired ripping production rate. “Very difficult ripping” refers to rocks in which the use of heavy construction equipment is likely to cease being a cost-effective method of excavation (necessitating the use of explosives to maintain a desired excavation rate). It should be emphasized that the cutoff velocities of this classification scheme are approximate and rock characteristics (such as fracture or joint spacing and orientation) play a significant role in determining rock rippability. These characteristics may also vary with location and depth in the rock mass.

In summary, the data indicates that the average seismic velocities of the near-surface bedrock materials along the seismic survey lines are approximately 1,900 feet per second. Below these near-surface materials (which are generally less than 1 to 7 feet in depth below the existing ground surface), the average seismic velocity of the slightly weathered to unweathered bedrock is 5,400 feet per second (with a range between approximately 4,000 and 6,500 feet per second). Below an estimated depth of approximately 15 to 25 feet, unweathered bedrock with average seismic velocities of approximately 11,000 feet per second (ranging between 8,560 and 13,771 feet per second) is anticipated.

Based on the results of the previous seismic refraction study and current rippability evaluation, it appears that the near surface granitic rock is generally rippable with heavy-duty construction equipment in good working order (i.e. a single shank D9R Dozer or equivalent) to depths on the order of 5 to 10 feet. However, moderately difficult ripping to very difficult ripping and localized blasting should be anticipated in localized areas within the upper 5 to 20 feet of the ground surface throughout the site. Below this point, the rock is expected to be unrippable with localized zones of marginally rippable granitic rock, becoming progressively more dense with increasing depth. Blasting is anticipated for planned excavations below a depth of 10 to 20 feet and for localized areas less than that depth. Localized residual boulders of dense rock are also anticipated within otherwise rippable zones.

A significant amount of rock including oversize material (i.e. rock typically over 8 inches in maximum dimension) will likely be generated during the grading of the site. The rock should be placed to prevent nesting of rocks, as recommended in Section 6.6 and Appendix F. In general because of the very dense nature of the rock at depth, we recommend preblasting of major cut areas prior to the removal of all over burden to help reduce the size of rock fragments and also to generate additional fines. Care will need to be exercised where blasting is performed near the planned face of slope. Excessive fracturing in the planned slope face may necessitate stabilization of areas that would have otherwise been stable without blast-induced fracturing.

Based on our review of the preliminary site development plans, areas of deep fill (i.e. greater than 50 feet) are proposed. In order to minimize the potential for differential settlement, fills deeper than 50 feet in depth should be placed at elevated moisture content and with increased compactive effort (95% relative compaction). In addition, canyon subdrains should also be provided in all major fill areas. All fills greater than 50 feet should be monitored for post construction settlements prior to the installation of settlement sensitive improvements.

4.3 Earthwork Shrinkage and Bulking

Based on the results of our investigation and our professional experience with similar projects in the general vicinity of the site, we have estimated bulking and shrinkage of the on-site soils. The volume change of excavated onsite materials upon recompaction as fill is expected to vary with materials and location. Typically, the surficial soils and bedrock materials vary significantly in natural and compacted density, and therefore, accurate earthwork shrinkage/bulking estimates cannot be determined. The following factors (based on the results of our investigation, geotechnical analysis and professional experience on nearby sites) are provided on Table 4 as guideline estimates. If possible, we suggest an area where site grades can be adjusted (during the later portion of the site grading operations) be provided as a balance area.

Table 4 Earthwork Shrinkage and Bulking Estimates	
Geologic Unit	Estimated Shrinkage/bulking
Undocumented Fill	5 to 15 percent shrinkage
Topsoil/Alluvium/Colluvium/Slopewash	0 to 10 percent shrinkage
Granitic Rock (weathered upper 2 to 10 feet)	0 to 8 percent bulking
Granitic Rock (unweathered rock below 2 to 10 feet)	15 to 25 percent bulking

5.0 CONCLUSIONS

Based on the results of our preliminary geotechnical investigation at the subject site and our review of the previous geotechnical reports, it is our opinion that the proposed development of the project is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications, and followed during site grading and construction.

The following is a summary of the geotechnical factors that may affect development of the site.

- Based on our subsurface exploration and review of pertinent geotechnical reports, the site is underlain primarily by Cretaceous Granitic Rock, with a thin veneer of compressible soils which include colluvium, topsoil, and undocumented fill soils. Locally, thicker surficial materials include alluvium and slopewash deposits.

- Geologic literature indicates the presence of possible ancient landslide debris on the eastern edge of the site and beneath Interstate 15 (SDAG, 1988). Based on our understanding of the proposed project, the mapped landslide is not in the vicinity of any proposed development.

- The undocumented fill, topsoil, colluvium, alluvium, slopewash, and weathered formational materials are porous and/or potentially compressible in their present state and will require removal and recompaction in areas of proposed development or future fill.

- It is anticipated that the surficial soils and upper highly weathered portion of the granitic rock may be excavated with conventional heavy-duty construction equipment. Localized areas within the upper portion of the site may require heavy ripping and/or localized blasting. Cut areas are expected to be only marginally rippable and will likely require generalized blasting. Blasting is anticipated below 10 to 20 feet of the surface and in outcrop areas.

- Care should be taken to avoid overblasting/overbreaking of cut slopes. Any loose material remaining on the cut slope faces should be scaled, by hand if necessary, in order to clean slope faces of loose, easily erodible material.

- The generation of a significant amount of rock (including oversized material) is anticipated during the grading of the site. Rock fills and oversized material should be placed in accordance with the recommendations presented in Section 6.6 and Appendix F.

- Areas of deep fill (i.e. greater than 50 feet) are proposed. In order to minimize the effects of potential differential settlement increased compaction and settlement monitoring is recommended for fills greater than 50 feet in depth

- The existing on-site soils are expected to be suitable material for use as artificial fill provided they are relatively free of organic material and debris. The Quaternary-aged alluvium, colluvium, and slopewash deposits are expected to provide granular fill material, with rare oversized clasts suspended with these deposits.

- Active faults are not known to exist on or in the immediate vicinity of the site. The main seismic hazard that may affect the site is from ground shaking from one of the active regional faults.

- Due to the high-density characteristics of the on-site bedrock materials and lack of a groundwater table, the potential for liquefaction in bedrock areas is considered low. The potential for liquefaction in alluvial areas is also considered low provided the alluvium is removed and replaced with compacted fill in areas of proposed grading/development.

- In general, when recompacted as fill soil, the surficial units (including topsoil, colluvium, alluvium, slopewash, etc.) are anticipated to shrink while the bedrock materials are likely to bulk.

- Natural slopes at the site contain local areas of potential surficial instability, including those identified on the Geotechnical Map (Plates 1 and 2) situated above areas of proposed development, mitigative measures may be required, as discussed in Section 6.7.2.

- It is anticipated that the proposed development will include cut slopes comprised of granitic rock. Provided adverse geologic conditions are not present, the cut slopes will not require stabilization measures to mitigate potential surficial instability concerns. In general proposed cut slopes may be constructed at gradings of 1-1/2:1

(horizontal to vertical). Localized areas of dense rock, such as along the entry road, may be stable at steeper gradings where free of adverse conditions. Mapping of all cut slopes should be performed during grading. If adverse geologic conditions (such as highly fractured and jointed rock, clay-lined fractures, seepage zones, etc.) are present, stabilization measures such as the placement of a stability fill or rock-bolting will be required. Slope stability recommendations are presented in Section 6.8.

- Foundation recommendations are not included as part of this report, but available once development plans, building types, and locations are finalized.

- Abandoned water wells may be present on site.

- An abandoned rock quarry is present on the site but is outside the areas proposed for development. Excavations on site however, may provide suitable material for use as road base and/or, aggregate, thus eliminating the need for importing of these materials. Additional testing and/or processing will be required for use of onsite materials.

6.0 RECOMMENDATIONS

6.1 Earthwork

Earthwork should be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix F. In case of conflict, the following recommendations shall supersede those in Appendix F. Also included are our typical details for Soil-Rock Fill, Utility Undercutting, and Building Pad Undercut in Bedrock Areas. The contract between the developer and earthwork contractor should be worded such that it is the responsibility of the contractor to place the fill properly and in accordance with the recommendations of this report and the specifications in Appendix F, notwithstanding the testing and observation of the geotechnical consultant.

6.2 Site Preparation

Prior to grading, the proposed residential and structural improvement areas of the site should be cleared of surface and subsurface obstructions, including any existing debris and undocumented or loose fill soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off site. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to above-optimum moisture conditions, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). Any water wells located within the areas of proposed improvements that do not remain in operation should be abandoned in accordance with County of San Diego Health Department guidelines.

6.3 Removal and Recompaction

The alluvium, topsoil, colluvium, slopewash, and undocumented fill soils that occur on site are potentially compressible in their present state and may settle under the surcharge of fills or foundation loadings. In areas that will receive additional fill soils that will support settlement-sensitive structures or other improvements (such as roadway utility lines, etc.), these soils should be removed down to competent material determined by the geotechnical consultant, moisture-conditioned, and recompacted to a minimum 90 percent relative compaction (based on ASTM

D1557) prior to placing fill. The removal limit should be established by a 1:1 projection from the edge of fill soils supporting settlement-sensitive structures downward and outward to competent material identified by the geotechnical consultant. Fill soils should be free of debris and organic materials (trees, shrubs, stumps, roots, leaves, and mulch derived from vegetation). As encountered in the trenches, the alluvium on site varied from 2 to 17+ feet in depth, although localized deeper areas may be encountered. The topsoil and colluvium across the site is generally on the order of 1 to 4 feet in depth. Review of the trench and boring logs (Appendix B) gives specific depths to competent formational materials in trench locations. Actual depths and limits of removals should be evaluated by the geotechnical consultant during grading.

Our experience in adjacent areas with similar conditions indicates that “hidden” areas of topsoils and colluvium should be expected on the lower portions of the sidehill areas. While soils are relatively thin in many areas, we expect that normal benching techniques may not be adequate during fill placement due to irregular topography and local deep pockets of soil. Some stripping of compressible soils therefore may be necessary on steep hillsides.

6.4 Excavations

Based on the results of our field investigations, we anticipate that in general, for most excavations between 5 to 10 feet in depth, areas of heavy ripping should be anticipated. Marginally rippable to non-rippable rock and localized residual boulders were also encountered in areas relatively near to the surface. The results of the rock drill and seismic traverse data are discussed in Section 4.2 and included as Appendices B and D, respectively. The trenches excavated across the site to evaluate the near surface soil conditions also give some data on site rippability (Appendix B).

It should be understood that fractures, bedding, joints, fractures, and weathered zones can greatly influence site rippability. The onsite bedrock consists of granitic rock. The fracture/jointing will greatly enhance the ability of well-maintained heavy equipment such as a Caterpillar D9R or D10R equipped with a single barrel ripper to excavate most of the onsite materials to 5 to 10 feet below the surface. It should be understood that dense layers and areas of non-rippable rock that require blasting should be anticipated. In addition, areas where confined excavations are proposed (such as trenches) may be difficult to excavate without blasting.

Numerous oversized boulders and surface exposures of non-rippable rock are present throughout the site. As a result, we anticipate that the excavation of otherwise rippable zones will generate significant amounts of oversize materials. Heavy ripping, jackhammering and blasting should be anticipated for major excavations and/or deep utilities. Because of the very dense nature of the granitic rock we recommend preblasting of major cut areas prior to the removal of all overburden in order to reduce the potential for fly rock and increase the effectiveness of blasting to generate fines and reduce rock fragment size. Care must be exercised where blasting is performed near planned slope faces. Over breaking/fracturing of slope faces may necessitate remedial measures that otherwise would not be necessary.

6.5 Structural Fills

The onsite soils are generally suitable for use as compacted fill provided they are free of organic materials and debris. Areas to receive structural fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to at least optimum moisture content, and recompacted to at least 90 percent relative compaction (based on ASTM D1557). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in thickness. Fill soils should be placed at a minimum of 90 percent relative compaction (based on ASTM D1557) at optimum or above optimum moisture content. Placement and compaction of fill should be performed in accordance with local grading ordinances under the observation and testing of the geotechnical consultant. Where fills are deeper than 50 feet, these materials should be compacted to 95 percent relative compaction.

Fills placed on slopes steeper than 5 to 1 (horizontal to vertical) should be keyed and benched into dense formational soils (see Appendix F for benching detail). Fills placed within 3 feet of finish pad grades should consist of granular soils of very low to low expansion potential and contain no materials over 8 inches in maximum dimension. Screening or crushing may be necessary to accomplish this recommendation. Oversize material may be incorporated into structural fills if placed in accordance with the recommendations in Section 6.6 below.

In addition, proposed cut building pads in rock areas should be undercut a minimum of 4 feet to facilitate foundation excavations and utility sweeps. Additional overexcavation may be needed if cut or fill transitions are greater than 20 feet. This should be reviewed as grading plans are finalized. Pad undercuts should slope toward the street or deep fill areas at a minimum of 1-2 percent to reduce the potential for the accumulation of water. For deep utilities we recommend the utility alignments (streets, etc.) be over-excavated a minimum of 1 foot below the deepest utility. Typical details for utility undercutting and pad undercutting in bedrock areas are included in Appendix F.

6.6 Rock Fill Specifications

We anticipate that relatively deep cuts and possibly shallow cuts on hilltops and ridges will generate oversized rock. In addition, minor amounts of oversize “float” rock is likely to be encountered within topsoil and colluvial layers and within otherwise rippable zones.

Within the upper 4 feet of finish grade of building pads and outer 5 feet of finish slope faces, fill soils should not contain rock greater than 8 inches in maximum dimension in order to facilitate foundation and utility trench excavation and to avoid slope surface irregularities. For fill soils between 4 and 8 feet below finish pad grade and between 5 and 15 feet of the slope face, the fill may contain rock up to 24 inches in maximum dimension and should be mixed with sufficient soil to eliminate nesting. We recommend that the owner consider surveying all building pads and fill slopes prior to placing the final capping fill soil containing rock less than 8 inches maximum dimension.

For street sections, the fill soils within the upper8 feet of the finish subgrade surface and at least 2 feet below the lowest pipes should not contain rock greater than 8 inches in maximum dimension. Below that rocks up to a maximum dimension of 36 inches may be incorporated into the fill utilizing rock blankets, as described in the following section 6.6.1. Rocks up to 5 feet in maximum dimension may also be placed in windrows in deeper fill areas in accordance with the section 6.6.2 below. Again, we recommend that the owner consider surveying all the street fills areas prior to placing the final fill soil containing rock less than 8 inches maximum dimension.

The finished slope should be track walked with a dozer to compact the slope face.. Typical soil-rock fill details are included within Appendix F. Boulders up to 10 feet in maximum dimension may be individually placed in non-structural fill.

Recommendations for an all rock fill can be provided on request, and may be deemed appropriate in certain areas as the proposed grading plans are further developed. Based on our review of the preliminary project plans provided, and the results of our investigation, we believe that construction of soil/rock fills, utilizing a combination of blankets and windrows, as described below, will provide the most feasible solution to site grading concerns. Modification to the above recommendations may be desired in areas if deep utility excavations or swimming pools are proposed.

6.6.1 Rock Blankets

Placement of artificial fill soils which include mixtures of sand, gravel, and abundant oversized material less than 36 inches in maximum dimension, can be accomplished utilizing rock blankets, consisting of horizontal layers of material placed on a properly conditioned surface (in accordance with the recommendations of Appendix F). Materials placed within 5 feet of a slope face should not exceed 8 inches in maximum dimension. The placement of rock lifts should be performed by rock trucks traversing the previously placed lift and dumping at the end of the previous-placed lift. Dozers shall be utilized to spread the rock and facilitate filling of any voids. Soil filling voids in rock fills should be flooded during placement with a sufficient amount of water to wash soil into all voids. Each lift should be thoroughly watered during placement with the water truck working in front of the rock lift being placed. Material filling voids should be compacted to a minimum of 90 percent, or 95 percent where fills are deeper than 50 feet.

6.6.2 Rock Windrows

Placement of larger rocks (up to 5 feet in diameter) is acceptable if done on an individual basis or within windrows and kept a minimum of 15 feet below finish grade. The base of the windrow should be excavated into the compacted fill core and rocks placed single file in the excavation. Sands and gravels should be added and thoroughly flooded and tracked until all voids are filled. Adjacent windrows should be separated by at least 15 feet horizontally and 4 feet vertically, in accordance with the recommendations of Appendix F.

6.7 Slope Stability

In our opinion, and based on our previous experience, within the development area major cut slopes within the dense granitic bedrock are proposed to maximum heights of roughly 85 feet at gradients of 1-1/2:1 (horizontal to vertical) or flatter and several 1:1 cut slopes in the proposed roadway. Based on our experience, these slopes are stable as designed provided they are free of adverse geologic conditions. In areas it may be desirable to redesign some of the proposed slopes in order to minimize grading of open space areas, and reduce rock quantities. Again, based on our preliminary calculations, these slopes (i.e., a generalized Cross-Section A-A') should be stable where free of adverse geologic conditions. Results of cut slope analysis performed using the GStabl 7 computer program are presented in Appendix E (Leighton, 2008).

Cut slopes should be geologically mapped during grading to evaluate the exposed conditions. Care should be taken not to overblast during excavation of proposed cut slopes. Some irregularities in the finish slope face are anticipated where a rock may locally protrude from the slope face or dislodging of a rock may result in a locally oversteepened condition. Care should be taken to not "paste" fill back into these areas.

As previously noted, we understand the project development plans have fill slopes proposed at inclinations of 1-1/2:1 (horizontal to vertical), or flatter, with maximum heights on the order of 185 feet. The alternative 1-1/2:1 fill slopes were analyzed for gross stability. (i.e.,generalized Cross-Sections B-B' and C-C'. Results of the static analysis are presented in Appendix E (Leighton, 2008). The strength parameters considered in our analyses are based on laboratory test results (Appendix C), our experience with similar units, and our professional judgment. The parameters utilized are as follows:

Table 5 Soil Strength Parameters			
Material	Unit Weight (pcf)	Cohesion (pcf)	Angle of Internal Friction (degrees)
Compacted Fill	135	150	40
Bedrock	140	1,000	42

Our previous analysis (Leighton, 2005a) indicates that the proposed cut and fill slopes, at the proposed heights and gradients, have calculated static factor of safety greater than 1.5 with respect to potential deep-seated failure. In our opinion, the proposed major cut slopes within the dense on site bedrock would be stable to the designed heights and gradients provided slopes are free of adverse geologic conditions. Based on our review of current grading (Fuscoe, 2015) plans, the designed cut and fill slopes are generally similar (i.e., slope gradients and geologic condition) to those on previous site development plans, and therefore are considered stable provided they are free of adverse geologic conditions.

Cut and fill slopes should be provided with appropriate surface drainage features and landscaped with drought-tolerant, slope-stabilizing vegetation as soon as possible after grading to reduce the potential for erosion. Berms should be provided at the top of fill slopes, and brow ditches should be constructed at the top of cut slopes. Lot drainage should be directed such that runoff on slope faces in minimized. Inadvertent oversteepening of cut and fill slopes should be avoided during fine grading and building construction. If seepage is encountered in slopes, special drainage features may be recommended by the geotechnical consultant.

We recommend against that exclusive use of generally cohesionless sand in the slope faces, as these materials are prone to erosive rilling. In addition, expansive clayey soils, if placed within 15 feet of the slope face, may be subject to surficial instability. We recommend that clayey soils be thoroughly mixed with poorly graded sands to produce better quality fill material which will be more effective in reducing erosion and increasing surficial stability.

6.7.1 Setback from Slopes

We recommend a minimum horizontal setback distance from the face of slopes for all structural foundations, footings, and other settlement-sensitive structures as indicated on the following table. This distance is measured from the outside bottom edge of the footing, horizontally to the slope or retaining wall face and is based on the slope or wall height. The foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 6

Minimum Foundation Setback from Slope Faces

Slope Height	Minimum Recommended Foundation Setback
less than 5 feet	7 feet
5 to 20 feet	10 feet
greater than 20 feet	H/2, where H is slope height; not to exceed 15 feet (10 feet maximum along cut slopes)

Please note that the soils within the structural setback area possess poor lateral stability, and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a pier and grade beam foundation system to support the improvement. The deepened footing should meet the setback as described above.

6.7.2 Natural Slope Stability

To increase Open Space areas relative to the proposed development, it is occasionally desirable to leave undisturbed natural slopes adjacent to development areas. Natural slopes on-site are subject to surficial instability, as indicated by the presence of slopewash deposits, source area scars, and perched granitic boulder outcrops, as shown on the Geotechnical Map (Plates 1 and 2). Such areas are of particular significance when located above and immediately adjacent to proposed development (Plates 1 and 2). Mitigative measures can be applied on a case-by-case basis, and include buffer areas without structural development, construction of debris walls, or catchment basins, or slope reconstruction and buttressing. The need for such mitigation should be based on review of final grading plans and field observations during grading.

6.7.3 Settlement

The varied site topography will result in a series of fill areas with differential fill thicknesses and depths. All fills deeper than 50 feet should be monitored for post construction settlement prior to construction of settlement sensitive improvements. Settlement can be monitored by the installation of a series of surface monuments installed as soon as the design grades have been achieved. Settlement monuments should be surveyed on two-week intervals until such time as the data indicates that the proposed construction can proceed. Estimates of anticipated settlement and potential construction delay can be provided after final grading plans and proposed building locations become available.

6.8 Slope Construction

6.8.1 Cut Slopes

In general it is anticipated that cut slopes over 10 feet will require blasting in order to excavate the slope. If rock debris remain on the 1:1 (horizontal to vertical) cut slopes, the cut rock slopes may need to be raked/scaled, with proper runoff control measures in-place. Cut slopes located within dense rock areas and free of adverse geologic conditions (jointing/fracturing/weathering) are anticipated to have an adequate factor of safety ($FS \geq 1.5$) for both deep seated and surficial stability. The upper portions of proposed cut slopes below open space areas will likely be weathered and may need to be replaced or laid back to a flatter gradient.

All cut slopes should be geologically mapped during construction. If areas of adverse conditions are identified, mitigation may be required. This may include removal of loose boulders or displaced blocks, possible stabilization fills, buttresses, or possibly the use of localized rock bolts and/or catchment netting. Some localized irregularities in the finish slope face of rock cut slopes should be anticipated.

6.8.2 Fill Slopes

Major fill slopes are proposed to heights on the order of 185 feet and gradients of 1-1/2:1 (horizontal to vertical). These slopes are considered to be grossly and surficially stable. Fill slopes should be provided with appropriate benches and drainage devices as recommended by the project civil engineer. In order to maintain an adequate factor of safety the material placed in these slope areas should consist of a uniform mixture of soil and rock fragments during grading to verify the strength parameters utilized in our analysis. Fill soils placed within half the slope height of major fills should conform to the following strength parameters. The static analysis is applicable for soils having an internal friction angle of at least 40 degrees and an apparent cohesion of at least 150 pounds per square foot.

Where fill slopes are proposed over natural ground the slopes should be properly keyed and benched in accordance with the recommendations of this report and the General Earthwork and Grading Specifications presented in Appendix F. Efforts should be made to avoid narrow “sliver fills.” To that end, all fill slopes should have a minimum width equivalent to at least one half the slope height. In addition, benches with a minimum width of at least 10 feet should be provided at 20 foot vertical intervals for all slopes over 50 feet in height. This may require the excavation of some non-rippable material.

Drains may be required for fill slopes keys. Also, for slopes in excess of 50 feet in height, and where the width of the fill does not exceed half of the slope height, horizontal back-drains should be provided on the benches at roughly 40-foot intervals. The need for such drains should be based on field observations during rough grading. Some of the onsite soils are easily eroded. Landscaping and drainage control devices should be installed as soon as practical to minimize surficial erosion.

6.9 Control of Ground Water and Surface Water

Our experience indicates that surface or near-surface ground water conditions can develop in areas where ground water conditions did not exist prior to site development, especially in areas where a substantial increase in surface water infiltration results from landscape irrigation. This sometimes occurs where relatively impermeable bedrock materials are overlain by granular fill soils. In addition, during slope excavations, seepage in cut slopes may be encountered. We recommend that an engineering geologist be present during grading operations to evaluate seepage areas. Drainage devices for reduction of water accumulation can be recommended when these conditions are observed.

In order to reduce the potential for ground water accumulation within fills, we recommend that canyon subdrains be installed in canyon areas beneath proposed fills. The subdrains should consist of a 6-inch diameter, perforated pipe placed in a shallow trench surrounded by filter materials as described in the attached General Earthwork and Grading Specifications (Appendix F). The actual subdrain location for each individual area should be evaluated during grading.

We recommend that measures be taken to properly finish grade each building area, such that drainage water from the building area is directed away from building foundations (2 percent minimum grade for a distance of 5 feet), floor slabs, and tops of slopes. Ponding of water should not be permitted, and installation of roof gutters which outlet into a drainage system is considered prudent. Planting areas at grade should be provided with positive drainage directed away from the building. Drainage and subdrain design for these facilities should be provided by the design civil engineer.

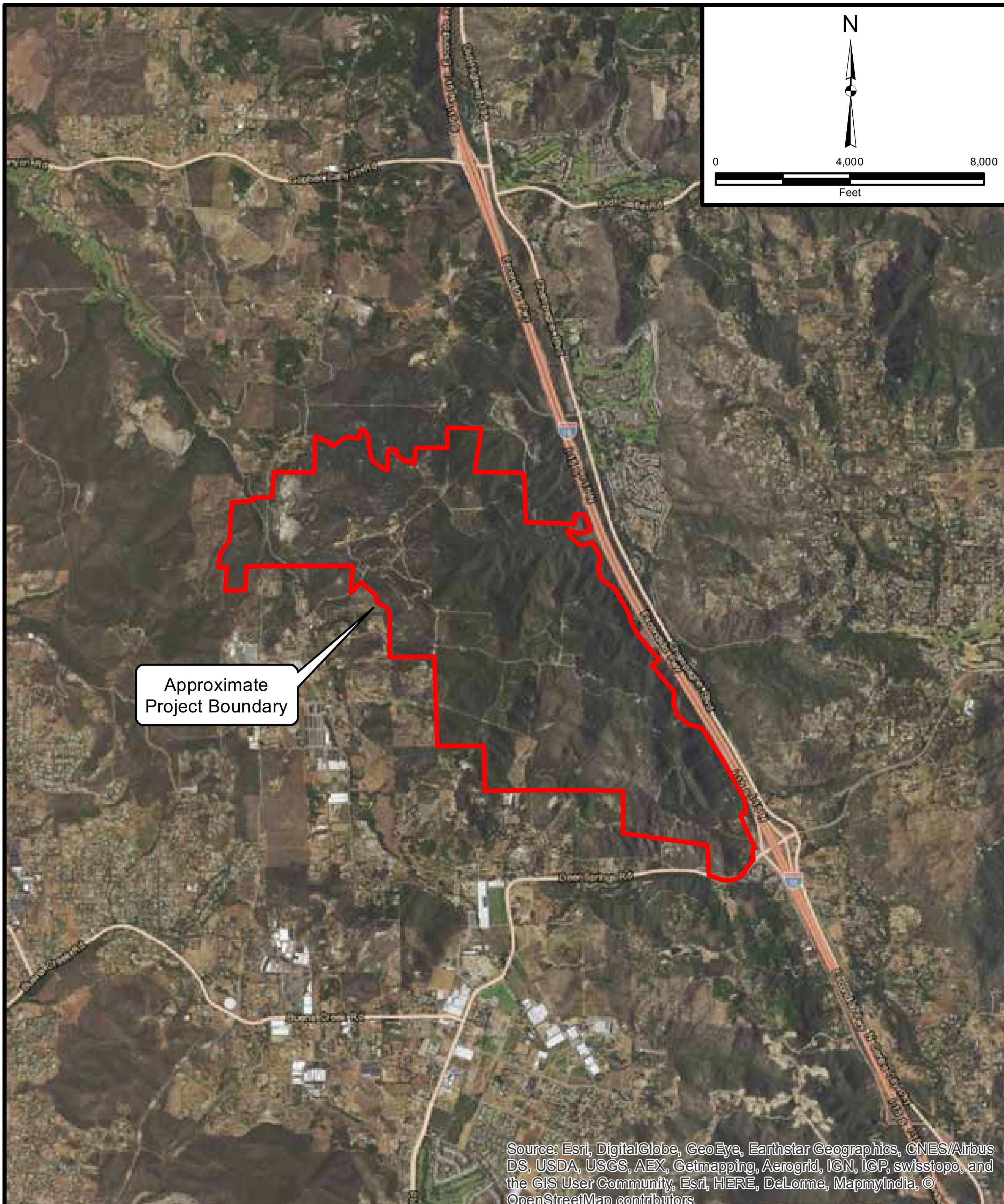
Localized seeps may also be encountered in cuts within the dense bedrock. Such seepage is generally controlled by fracturing or bedrock jointing. If encountered during grading, mitigative measures can be recommended by the geotechnical consultant. These recommendations will likely include additional subdrains and possible overexcavation of building areas. It is possible that after site development is complete, some additional zones of seepage may occur due to site irrigation in areas with no previous history of seepage. Recommendations for the treatment of these areas can be provided on an individual basis.

Where desilting basins are proposed, seepage may occur along the outflow structure. To minimize the potential for seepage in this area, we recommend that cut-off-walls be incorporated into the design. Cut off walls should have a minimum width and extend for at least 6 inches beyond the sides of the trench excavated for the outlet pipe. Cut off walls should extend at least 24 inches above the top of pipe and consist of poured-in-place concrete.

7.0 LIMITATIONS

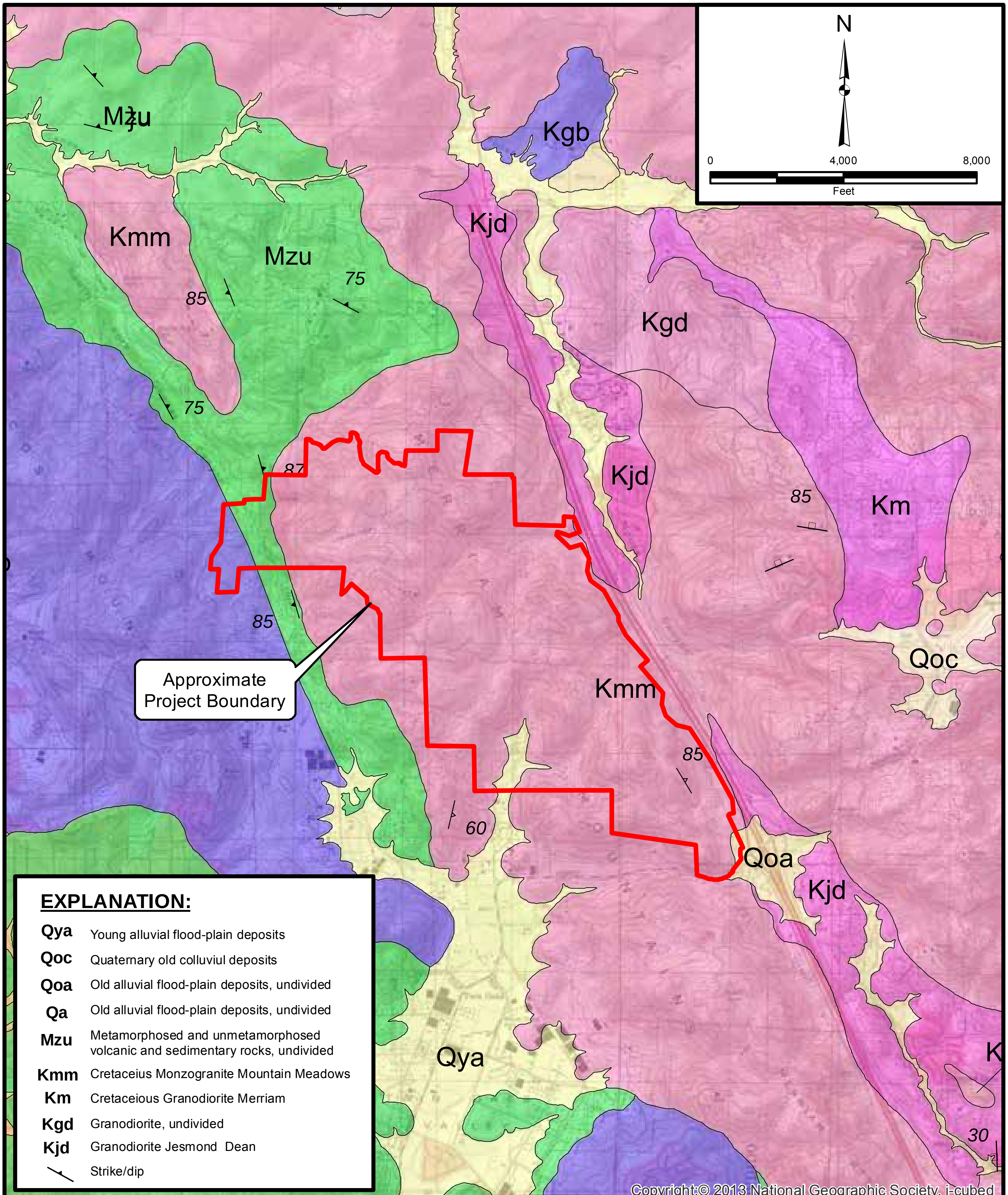
The findings, conclusions and recommendations in this report are based in part upon data that were obtained from widely spaced subsurface investigations and limited geotechnical analysis. Such information is by necessity incomplete. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to review final grading plans and to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

FIGURES AND PLATES



Project: 10618.002	Eng/Geol: WDO/RCS	SITE LOCATION MAP	Figure 1
Scale: 1 " = 4,000 '	Date: November 2015	Newland Sierra	
Base Map: ESRI ArcGIS Online 2015		San Diego, California	Leighton
Author: mmurphy (mmurphy)			

Map Saved as V:\Drafting\10618\002\Maps\10618-002_F01_SLM_2015-06-26.mxd on 6/26/2015 4:02:13 PM



Project: 10618.002	Eng/Geol: WDO/RCS	REGIONAL GEOLOGY MAP	Figure 2
Scale: 1 " = 4,000 '	Date: November 2015	Newland Sierra San Diego, California	Leighton

Geoogy map: Map of the Oceanside 30'x60' quadrangle, California, compiled by Michael P. Kennedy and Siang S. Tan, 2008

Author: mmurphy (mmurphy)

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Figure P1: Weakly jointed granite (Kgr) near the surface, characteristic ridgeline exposure.

Figure P2: Weakly jointed granite (Kgr), with minimal topsoil, characteristic ridgeline exposure.

Figure P3: Alluvium (Qal) (generally light colored grass area) within southern canyon.

Figure P4: Quaternary Slopewash (Qsw) exposed within erosional gully.

Figure P5: Quaternary slopewash (Qsw) exposed within erosional gully.

Figure P6: Granite boulders perched on natural hillslope, representative view.

Figure P7: Granite boulders at base of sloped area.

Figure P8: Moderately jointed granite, with thin colluvium development.


PLATE 1
GEOTECHNICAL MAP
Newland Sierra
San Diego, California
Leighton
Proj: 10618.002
Eng/Geol: WDO/RCS
Scale: 1"=250'
Date: November 2015
Drafted By: MAM
Checked By:
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MATCHLINE
N
LEGEND
GEOLOGIC SYMBOLS
Afu ARTIFICIAL FILL, UNDOCUMENTED
Qal QUATERNARY ALLUVIUM
Qalo OLDER QUATERNARY ALLUVIUM (CIRCLED WHERE BURIED)
Qsw QUATERNARY SLOPEWASH DEPOSITS
Qls? POSSIBLE QUATERNARY LANDSLIDE DEPOSITS, QUERIED WHERE UNCERTAIN
Kgr CRETACEOUS GRANITICS, CIRCLED WHERE BURIED
KJm JURASSIC–CRETACEOUS METAVOLCANIC ROCK
BOULDER FIELDS SITUATED ON STEEP (APPROXIMATELY 2:1(H:V)) ESCARPMENTS, ARROW INDICATES DOWNWARD DIRECTION
AREA OF SURFICIAL FAILURE, DEBRIS FLOW SCAR/SOURCE AREA
F INFERRED TREND OF FRACTURE (DIKE/JOINT) FABRIC
60 N50E MAPPED JOINT ATTITUDE
N45E VERTICALLY ORIENTED JOINT ATTITUDE
?. APPROXIMATE GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN, DOTTED WHERE BURIED
OTHER SYMBOLS
T–48 APPROXIMATE LOCATION OF EXPLORATORY TRENCH, PREVIOUS INVESTIGATION (EXCAVATED, WITH JD 710 BACKHOE TO PRACTICAL REFUSAL EXCEPT WHERE NOTED)
S–13 APPROXIMATE LOCATION OF SEISMIC TRAVERSE, PREVIOUS INVESTIGATION
FP–4 APPROXIMATE LOCATION OF FIELD PERCOLATION TEST
AT–44 T.D.45' APPROXIMATE LOCATION OF AIR–TRACK BORING, THIS INVESTIGATION (T.D.=TOTAL DEPTH)
BH–3 T.D.20' APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORING (T.D.=TOTAL DEPTH)
T–100 APPROXIMATE LOCATION OF EXCAVATOR TRENCH/PIT, THIS INVESTIGATION (EXCAVATED WITH EX450 EXCAVATOR TO PRACTICAL REFUSAL EXCEPT WHERE NOTED)
S–18 APPROXIMATE LOCATION OF SEISMIC TRAVERSE, THIS INVESTIGATION
T–7 APPROXIMATE HAND EXCAVATED TEST PITS, SMITH PROPERTY (LEIGHTON)
SITE BOUNDARY